FALL 2018 HOMEWORK 3 CEE 141

DUE: Monday, October 29th

NOTE: Perform calculations for LRFD only.

I. Complete the following problems from the textbook:

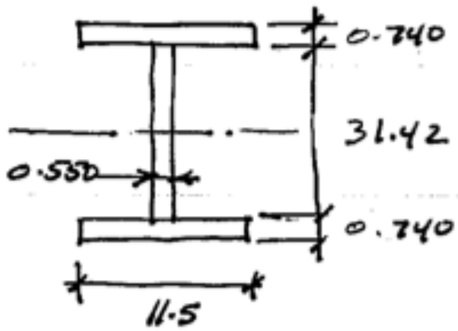
Chapter 6 – Bending Members

6-3, 6-9, 6-22, 6-34, 6-40, 6-44, 6-47, 6-49, 6-51

6-3

3. W33×118 $d = 32.9$ in, $t_w = 0.550$ in, $b_f = 11.5$ in., $t_f = 0.740$ in.

$S_x = 359$ in.^{3}, $Z_x = 415$ in.^{3}



$$I_x = \frac{11.5(32.9)^3}{12} - \frac{(11.5-0.550)(31.42)^3}{12}$$

$$= 5820 \text{ in.}^4$$

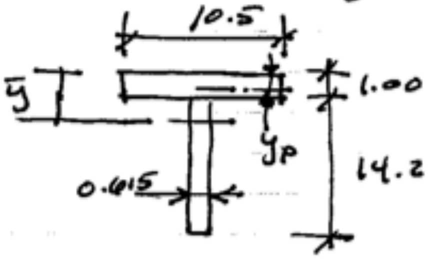
$$S_x = \frac{5820}{(32.9/2)} = \underline{354 \text{ in.}^3}$$

$$Z_x = 2\left[11.5(0.740)\left(\frac{0.740}{2} + \frac{31.42}{2}\right) + \frac{31.42}{2}(0.550)\left(\frac{31.42}{4}\right)\right]$$

$$= \underline{409 \text{ in.}^3}$$

6-9

9. WT 15 x 66 $d = 15.2\text{ in.}, t_w = 0.615\text{ in.}, b_f = 10.5\text{ in.}, t_f = 1.00\text{ in.}$
$\bar{y} = 3.90\text{ in}, S_x = 37.4\text{ in}^3, y_p = 0.921, Z_x = 66.8\text{ in}^3$



$A = 10.5(1.00) + 14.2(0.615) = 19.23\text{ in}^2$

Elastic Neutral axis

$$\bar{y} = \frac{10.5(1.00)(0.5) + 14.2(0.615)\left(1.0 + \frac{14.2}{2}\right)}{19.23}$$

$$= \underline{3.95}\text{ in.}$$

$$I_x = \frac{10.5(1.00)^3}{12} + 10.5(1.00)\left(3.95 - \frac{1.0}{2}\right)^2$$

$$+ \frac{0.615(14.2)^3}{12} + 14.2(0.615)(11.25 - 7.10)^2$$

$$= 126 + 297 = 423\text{ in.}^4$$

$$S_x = \frac{423}{11.25} = \underline{37.6\text{ in.}^3}$$

Plastic Neutral Axis

$$A = 19.23\text{ in.}^2 \quad \frac{A}{2} = \frac{19.23}{2} = 9.615\text{ in.}^2$$

if y_p is in flange

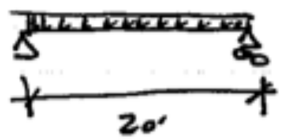
$$y_p = \frac{9.615}{10.5} = \underline{0.92\text{ in.}}$$ Thus in flange

$$Z_x = 10.5(0.92)\left(\frac{0.92}{2}\right) + 10.5(0.08)\left(\frac{0.08}{2}\right)$$

$$+ 14.2(0.615)\left(0.08 + \frac{14.2}{2}\right) = \underline{67.1}\text{ in.}^3$$

6-22

22. SELECT Beam $w_D = 0.8 \text{ k/ft}, w_L = 2.3 \text{ k/ft}$



Consider yielding only

a) LRFD $w_u = 1.2(0.8) + 1.6(2.3) = 4.64 \text{ k/ft}$

$$M_u = \frac{4.64(20)^2}{8} = 232 \text{ ft-k}$$

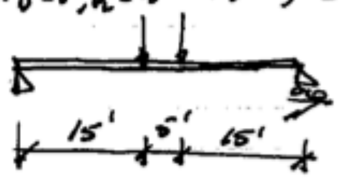
$$Z_{req} = \frac{232(12)}{0.9(50)} = 61.87 \text{ in}^3$$

Using Table 3-2 select __W18x35__, $Z_x = 66.5 \text{ in}^3$

6-34

34. Select a beam

$P_D = 8^k, P_L = 18^k \quad P_D = 4^k, P_L = 9^k$

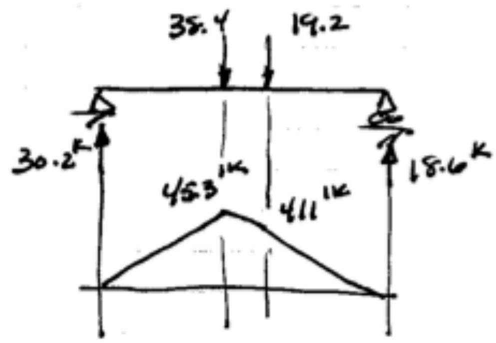


$C_b = 1.0 \quad L_b = 15', 5', 15'$

a) LRFD

$P_{u_L} = 1.2(8) + 1.6(18) = 38.4^k$

$P_{u_R} = 1.2(4) + 1.6(9) = 19.2^k$



$M_u = 453^{1k}, L_b = 15', C_b = 1.0$

Table 3-10 select W24x68

$\phi M_n = 486^{1k}$

$M_{self} = \dfrac{1.2(0.068)(35)^2}{8} = 12.5^{1k}$

Assume the self weight moment, max at 17.5' is added to max moment at left load. This is conservative

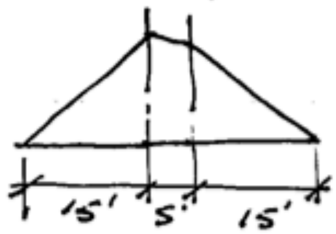
$M_u = 453 + 17.5 = 471^{1k} < \phi M_n = 486^{1k}$

$\therefore$ USE W24x68

6-40

40. Redesign Prob 34 WITH correct C_b

Moment diagram from prob 34



USE $C_b = 1.67$ for $L_b = 15$ ft.

the uniform self weight will be ignored for C_b.

a) LRFD

use self weight from prob 34 $\omega_{self} = 68^{\#}/ft$

$M_u = 466^{1k}$, $L_b = 15'$, $C_b = 1.67$

ENTER TABLE 3-10 with $\frac{M_u}{C_b} = \frac{466}{1.67} = 279^{1k}$

Try W18x55 $\phi M_p = 420 < 466$ ∴ no good

Use TABLE 3-2 Select W21x55, $\phi M_p = 473^{1k} > 466^{1k}$

and

$$\phi M_n = 1.67\left[473 - 13.8(15-5.9)\right] = 580 > 473$$

$\therefore \phi M_n = 473^{1k} > 466^{1k}$

∴ USE W21x55

6-44

44. W27x84, A992 & A913 Gr 65

$Z_x = 244 \text{ in.}^3 \quad S_x = 213 \text{ in.}^3 \quad \frac{b_f}{2t_f} = 7.78 \quad h/t_w = 52.7$

This shape is compact for both steels

A992 $\quad L_p = 7.31' \quad L_r = 20.8'$ (Table 3-2)

$$M_p = \frac{50(244)}{12} = 1020^{1k} \quad M_r = \frac{0.7(50)(213)}{12} = 612^{1k}$$

$$BF = \frac{1020 - 612}{20.8 - 7.31} = 30.2^k$$

A913 Gr 65, $r_y = 2.07$, $r_{ts} = 2.54$

$$L_p = \frac{1.76(2.07)}{12}\sqrt{\frac{E}{65}} = 6.41 \text{ ft}$$

$$L_r = \frac{1.95(2.54)}{12}\left(\frac{29000}{0.7(65)}\right)\sqrt{\frac{2.81(1.0)}{213(26.1)} + \sqrt{\left(\frac{2.81(1.0)}{213(26.1)}\right)^2 + 6.76\left(\frac{0.7(65)}{29000}\right)^2}}$$

$= 17.9$ ft.

$$M_p = \frac{65(244)}{12} = 1320^{1k}$$

$$M_r = \frac{0.7(65)(213)}{12} = 808^{1k}$$

$$BF = \frac{1320 - 808}{17.9 - 6.41} = 44.6^k$$

For $L_b = 18$ ft.

A992 $\quad M_n = 1020 - 30.2(18.0 - 7.31) = 697^{1k}$

A913 Gr 65

$$M_n = 1320 - 44.6(18.0 - 6.41) = 803^{1k}$$

6-47

47. Available shear W 30x90 (see user note in Sect G2)

$$h/t_w = 57.5 > 2.24\sqrt{\frac{E}{F_y}} = 53.9$$

$$\therefore \phi = 0.9 \; , \; \Omega = 1.67$$

$$h/t_w = 57.5 < 1.10\sqrt{\frac{E}{F_y}} = 59.2$$

$$\therefore C_v = 1.0$$

$$V_n = 0.6 F_y A_w = 0.6(50)(29.5)(0.47) = 416^k$$

a) LRFD

$$\phi V_n = 0.9(416) = 374^k$$

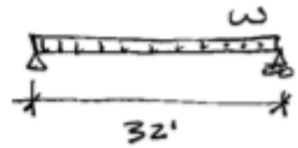
same as Table 3-2

6-49

49. DESIGN BEAM

$w_D = 2.3^{K}/ft + \text{self weight}$

$w_L = 3.1^{K}/ft.$



$C_b = 1.14$ (TABLE 3-1)

a) LRFD

$w_u = 1.2(2.3) + 1.6(3.1) = 7.72^{K}/ft.$

$V_u = \frac{32}{2}(7.72) = 124^{K}$

$M_u = \frac{7.72(32)^2}{8} = 988^{1K}$

ENTER TABLE 3-10 WITH $\frac{M_u}{C_b} = \frac{988}{1.14} = 867^{1K}$

TRY W24x146 $\phi M_p = 1570^{1K} > 988 \therefore$ ok

Add self weight

$M_{u_{self}} = \frac{1.2(0.146)(32)^2}{8} = 22.4^{1K}$

$M_u = 988 + 22.4 = 1010^{1K}$ $\frac{M_u}{C_b} = \frac{1010}{1.14} = 886^{1K}$

From Table 3-10 W24x146 still works and $\phi M_p > 886^{1K}$

Check Shear

Table 3-2 $\phi V_n = 482^{K} > 124 + \frac{1.2(0.146)(32)}{2} = 127^{K}$

$\therefore$ ok

Check Deflection

$\Delta = \frac{5}{384}\frac{(3.1)(32)^4(1728)}{(29000)(4580)} = 0.55 \text{ in.} < \frac{L}{360} = \frac{32(12)}{360} = 1.07 \text{ in.}$

$\therefore$ USE W24x146

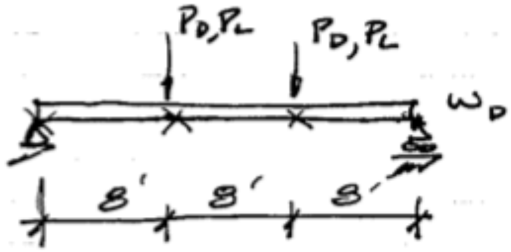
6-51

51. DESIGN BEAM

$w_D = 1.1^{k}/ft$

$P_D = 3.4^{k}$

$P_L = 6.0^{k}$



$L_b = 8 ft.$

Based on Moment between concentrated loads conservatively use $C_b = 1.0$ (calculates to $C_b = 1.01$)

a) LRFD

$w_u = 1.2(1.1) = 1.32^{k}/ft$

$P_u = 1.2(3.4) + 1.6(6.0) = 13.7^{k}$

$$M_u = \frac{1.32(24)^2}{8} + 13.7(8) = 205^{k}$$

$$V_u = \frac{1.32(24)}{2} + 13.7 = 29.5^{k}$$

TABLE 3-10, $L_b = 8.0$ ft.

Try W16x36 $\phi M_n = 215^{k}$

Check shear. (TABLE 3-2)

$\phi V_n = 141^{k} > V_u = 29.5$

CHECK DEFLECTION

$$\Delta = \frac{PL^3}{28EI} = \frac{6.0(24)^3(1728)}{28(29000)(448)} = 0.39 \text{ in.} < \frac{L}{360} = \frac{24(12)}{360} = 0.8 \text{ in.}$$

∴ USE W16x36

II. Answer the following problems:

1. The beam in Figure 1 is a W10x77 and has continuous lateral support. The load P is a service live load. A992 steel is used. What is the maximum permissible value of P?

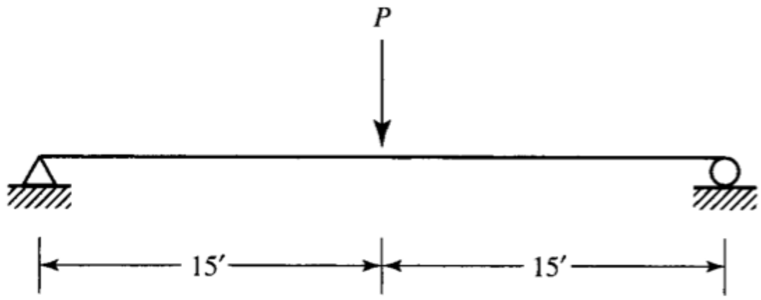


Figure 1

→ Check for compactness. From Part 1 of the Manual,

$$\frac{b_f}{2t_f} = 5.86 < 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{50}} = 9.15$$ → the flange is compact.

$$\frac{h}{t_w} = 14.8 < 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29{,}000}{50}} = 90.55$$ → the web is compact.

→ A W10x77 is compact for $F_y = 50$ ksi
(Note that there is no footnote in the properties tables to indicate that the section is not compact.)

→ From setting demand equal to capacity (beam has continuous lateral support, so no LTB issue)

$$M_u \le \varphi_b M_n$$

$$\frac{P_u L}{4} \le 0.9F_y Z_x = 366\ kip-ft$$

$$P_u \le 366\ kip-ft * 4\ /\ 30\ ft$$

$P_u \le 48.8\ kips$ → with load combination 1.2D+1.6L, $P_L \le 48.8$ kips/ 1.6 → **$P_L \le 30.5$ kips**

→ From considering deflection criteria of $\Delta_L < L/360$:

$$\frac{P_L L^3}{48EI} \le \frac{L}{360} = \frac{30 * 12"}{360} = 1"$$

$$P_L \le \frac{L}{360} * \frac{48EI}{L^3} = 1" * \frac{48 * 29{,}000\ ksi * 455\ in.^3}{(30 * 12")^3} = 13.575\ k$$ → $\boldsymbol{P_L \le 13.6\ kips}$

2. The beam shown in Figure 2 is a two-span beam with a pin (hinge) in the center of the left span, making the beam statically determinate. There is continuous lateral support. The concentrated loads are service live loads. Determine whether a W18x60 of A992 steel is adequate.

Verify that this shape is compact. For the flange,

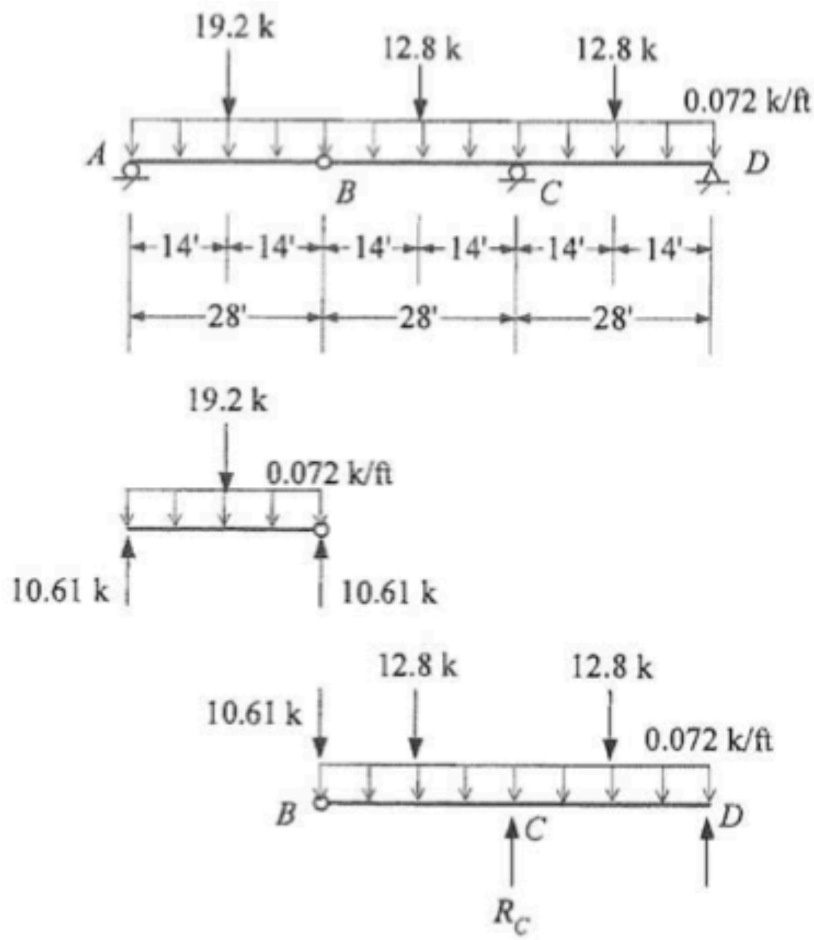
$$\lambda_p = 0.38\sqrt{\frac{29{,}000}{50}} = 9.15, \qquad \lambda = \frac{b_f}{2t_f} = 5.44 < \lambda_p \quad \therefore \text{ flange is compact}$$

$\frac{h}{t_w} < 3.76\sqrt{\frac{E}{F_y}}$ (for all shapes in the Manual for $F_y \leq 65$ ksi), so the web is compact, and the shape is compact.

$$M_n = F_y Z_x = \frac{50(123)}{12} = 512.5 \text{ ft-kips}$$

(a) Factored loads, including beam weight:

$$1.2(0.060) = 0.072 \text{ kips/ft}, \qquad 1.6(12) = 19.2 \text{ kips}, \qquad 1.6(8) = 12.8 \text{ kips}$$

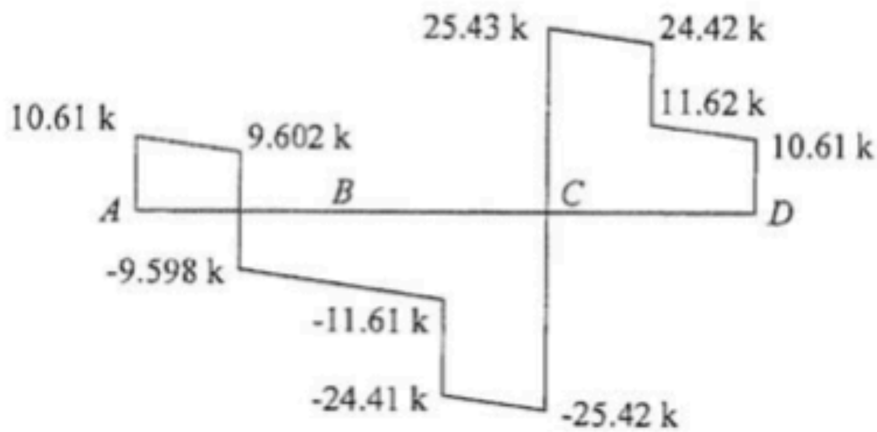


Reaction at C :

$$\sum M_D = -10.61(56) - 12.8(42) - 12.8(14) + R_C(28) - 0.072(56)^2/2 = 0$$

$$R_C = 50.85 \text{ kips}$$

Shear diagram:



Maximum moment occurs at C. Using the free-body diagram of BCD,

$$M_C = -10.61(28) - 12.8(14) - 0.072(28)^2/2 = -505 \text{ ft-kips}$$

Design strength $= \phi_b M_n = 0.90(512.5) = 461$ ft-kips

$M_u = 505$ ft-kips $> \phi_b M_n = 461$ ft-kips (N.G.) A W18 × 60 is not adequate.